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SCE&G
A SCANA COMPANY

March 12, 2009
NND-09-0048

U.S. Nuclear Regulatory Commission
Document Control Desk
Washington, DC 20555

ATTN: Document Control Desk

Subject: Virgil C. Summer Nuclear Station (VCSNS) Units 2 and 3 Combined License Application (COLA) - Docket Numbers 52-027 and 52-028 Response to NRC Request for Additional Information (RAI) Letter No. 031

Reference: Letter from Ravindra G. Joshi (NRC) to Alfred M. Paglia (SCE&G), Request for Additional Information Letter No. 031 Related to SRP Section 2.5.4 for the Virgil C. Summer Nuclear Station Units 2 and 3 Combined License Application, dated February 10, 2009.

The enclosure to this letter provides the South Carolina Electric & Gas Company (SCE&G) response to the RAI items included in the above referenced letter. The enclosure also identifies any associated changes that will be incorporated in a future revision of the VCSNS Units 2 and 3 COLA.

The responses to NRC RAI Numbers 02.05.04-3, 02.05.04-4, 02.05.04-6, 02.05.04-8, 02.05.04-9 and 02.05.04-11 are still under development and review by SCE&G. The final responses to those RAIs are expected be provided to the NRC by March 31, 2009.

Should you have any questions, please contact Mr. Al Paglia by telephone at (803) 345-4191, or by email at apaglia@scana.com.

I declare under penalty of perjury that the foregoing is true and correct.

Executed on this 12th day of March, 2009.

Sincerely,

Ronald B. Clary
General Manager
New Nuclear Deployment

D083
NRO

JMG/RBC/jg

Enclosure

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NRC RAI Letter No. 031 Dated February 10, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-1

FSAR Subsection 2.5.4.2.5.3 (pg 2.5.4-13) states that an N60 value of 30 blows per foot (bpf) was selected for the compacted structural fill, and that this value was deemed to be reasonable and conservative. However, the basis for selecting this specific value is not clear.

In order for the staff to assess whether 30 bpf is a reasonable and conservative value for the compacted structural fill, please explain the basis for selecting this numerical value.

VCSNS RESPONSE:

As noted in FSAR Subsection 2.5.4.5.3.1, the structural fill material that forms the basis of the N_{60} = 30 bpf is a granitic sand produced from crushing operations at Martin Marietta Aggregate's North Columbia Quarry, about 20 miles from the site. Two bulk samples were randomly selected from the quarry's stockpile. Grain size tests show one sample (MM-1) to have a USCS classification of SW with 3% fines, and the other (MM-2) to be SW-SM, with 10% fines. The samples are medium to coarse grained. The coefficient of uniformity of these samples is 10 and 18, respectively. (The grain size test results are shown on FSAR Figure 2.5.4-234).

As would be expected with such a well-graded material, the maximum density when compacted is high. Modified Proctor (ASTM D 1557) results on the two samples are shown on FSAR Figure 2.5.4-235. Maximum modified Proctor dry density is 122.9 pcf for sample MM-1 at 10.7% optimum moisture content. The corresponding values for sample MM-2 are 125.2 pcf and 8.2%. As noted in FSAR Section 2.5.4.2.5.3, this material will be compacted to at least 95% of the modified Proctor dry density in the field. This would result in in-situ total densities of at least 129 pcf at optimum moisture content, which is a high density for compacted sand.

Although no direct correlation exists between compaction expressed in terms of modified Proctor dry density, and compaction expressed in terms of relative density, when a high degree of compaction is required for granular fill in the field, at least 95% modified Proctor dry density and at least 70% relative density are specified. Thus, the actual densities obtained using these two criteria will be similar. The attached table (from USACE, 1992) shows approximate correlations between relative density and (1) angle of internal friction and (2) N_{60} for sands. For a well-graded medium to coarse sand with a relative density of 70%, the angle of internal friction from Table (a) is in the

41 to 44 degree range. For a uniform fine-grained sand, the corresponding friction angle is in the 36 to 39 degree range, i.e., the coarser more well-graded sand has a higher friction angle (and thus a higher N-value). Table (b) shows that at 70% relative density, the average N_{60} value expected is 40 bpf. It is expected that the value for fine-grained uniform sands will be less than 40 bpf, and for coarser grained well-graded sands will be more than 40 bpf.

In summary, the fill material tested is a well-graded medium to coarse sand that, when compacted to at least 95% modified Proctor compaction in the field will give a total density of at least 129 pcf, based on laboratory compaction test results. As expected for this material, this is a high value. Using the approximate correlation between 95% modified Proctor compaction and 70% relative density compaction, the literature indicates an internal friction angle of between 41 and 44 degrees can be expected, and the N_{60} value will be more than 40 bpf. Thus the N_{60} value of 30 bpf assumed in the FSAR is conservative.

Effective Angle of Internal Friction of Sands

a. Relative Density and Gradation
(Data from Schmertmann 1978)

Relative Density D_r, Percent	Fine-Grained		Medium-Grained		Coarse-Grained	
	Uniform	Well-Graded	Uniform	Well-Graded	Uniform	Well-Graded
40	34	36	36	38	38	41
60	36	38	38	41	41	43
80	39	41	41	43	43	44
100	42	43	43	44	44	46

b. Relative Density and In Situ Soil Tests

Soil Type	Relative Density D_r, Percent	Standard Penetration Resistance N_{60} (Terzaghi and Peck 1967)	Cone Penetration Resistance q_c, ksf (Meyerhof 1974)	Friction Angle ϕ', deg		
				Meyerhof (1974)	Peck, Hanson and Thornburn (1974)	Meyerhof (1974)
Very Loose	< 20	< 4	—	< 30	< 29	< 30
Loose	20–40	4–10	0–100	30–35	29–30	30–35
Medium	40–60	10–30	100–300	35–38	30–36	35–40
Dense	60–80	30–50	300–500	38–41	36–41	40–45
Very Dense	> 80	> 50	500–800	41–44	> 41	> 45

References:

USACE (1992). U.S. Army Corps of Engineers. Bearing Capacity of Soils, EM-1110-1-1905, Washington, DC.

Meyerhof, G.G.(1974). "Ultimate Bearing Capacity of Footings on Sand Overlying Clay," *Canadian Geotechnical Journal,* Volume 11, pp 223-229.

Peck, R.B., Hanson, W.E., and Thornburn, T.H. (1974). Foundation Engineering, John Wiley and Sons Ltd., New York.

Schmertmann, J.H. (1978). Guidelines for Cone Penetration Test Performance and Design, Report No. FHWA-TS-78-209., FHWA. McLean, VA.

Terzaghi, K., and Peck, R.B. (1967). Soil Mechanics in Engineering Practice, 2nd Edition, John Wiley and Sons, ltd, New York.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 10, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-2

FSAR Subsection 2.5.4.2.5.4 (pg 2.5.4-13 and Figure 2.5.4-218) presents the results of 3 RCTS tests, one for saprolite and 2 for proposed fill materials. Regulatory Guide 1.138 states that a sufficient number of tests should be performed to cover the range of values expected under field conditions for the material properties being assessed. However, variations in test results due to differences in materials being tested or to scatter resulting from testing techniques may not be apparent with this small number of tests. Also, the number of tests may not be sufficient for evaluating the validity of the test results.

In order for the staff to evaluate the comparison of RCTS results with the generic curves used in the seismic soil column analyses, shown in FSAR Figure 2.5.4-218, please justify the use of the generic curves based on such sparse site-specific data and explain the apparent divergence between test results and certain of the curves.

VCSNS RESPONSE:

RCTS tests were performed on two materials, namely offsite sand that is proposed for use as structural fill, and the in-situ saprolitic soils.

<u>Sand Fill</u>

As noted in the response to RAI 02.05.04-1, the proposed structural fill is a clean well-graded granitic sand, produced during crushing operations at a quarry about 20 miles from the site. Two random bulk samples of the material produced very similar grain size and modified Proctor curves, as would be expected from a material that is produced under controlled conditions.

The RCTS tests performed on each sample also produced very similar results, as shown in FSAR Figures 2.5.4-218b and 2.5.4-218c. Figures 2.5.4-240b and 2.5.4-240c show the Figure 2.5.4-218 results with the generic EPRI sand curve used in the SHAKE analysis superimposed. For both samples, the RCTS test results for G/G_{max} versus shear strain are in good agreement with the generic curve, and are close to a best fit line. The generic EPRI sand curve for material damping (D) is very close to a best fit curve for the RCTS D versus shear strain results for sample MM-1. The same is true for sample MM-2 except the torsional shear and resonant column test results diverge at higher strains, and the generic curve best fits the torsional shear results.

In summary, all of the test results indicate, as expected, that the proposed structural fill material is very consistent. The RCTS tests on samples of the fill agree well with the generic EPRI sand curves used in the SHAKE analysis for the fill column beneath the annex building. Since this fill will not be used beneath any seismic Category I structure, two RCTS tests on this material are considered to be sufficient.

Saprolite

As noted in FSAR Section 2.5.4.8, the loading from the seismic Category I nuclear island is transferred entirely to the underlying sound rock, or to concrete fill between the rock surface and the underside of the foundation. For the other major structures, including the seismic Category II annex building, structural fill extends below the structures to the underlying rock. The structural fill extends laterally so that no load is transmitted to the in-situ saprolitic soils or the saprolite common fill. Thus, the purpose of the field and laboratory testing of the saprolite is to complete the characterization of the site – the results do not impact the stability of the structures.

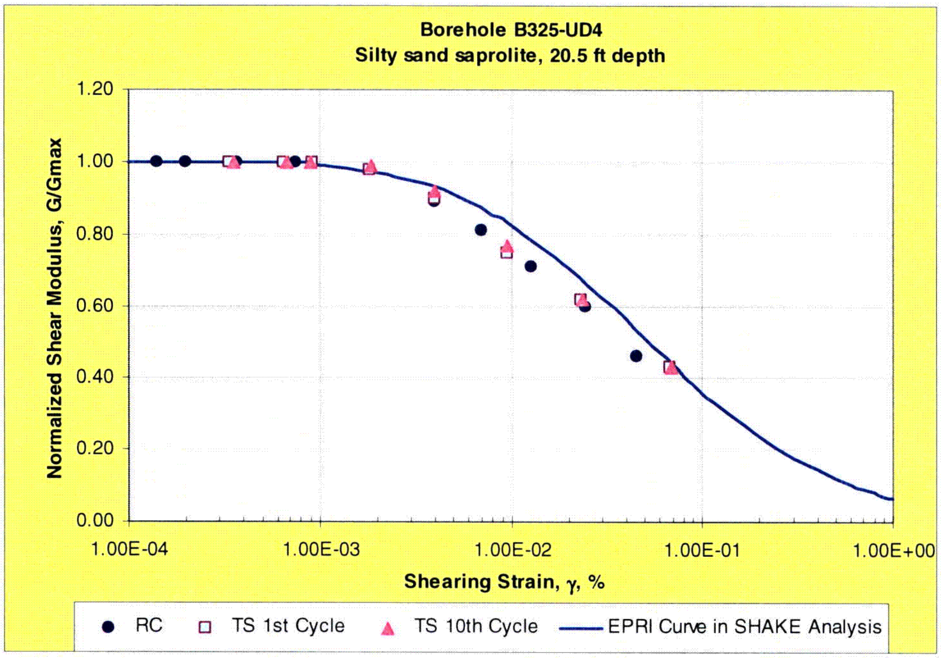
Included in the site characterization is the assessment of the liquefaction potential of the in-situ saprolitic soils. To obtain the peak ground acceleration in these soils, the SHAKE analysis required G/G_{max} and D versus shear strain curves for the in-situ saprolite soil column. The SHAKE analysis was run using generic EPRI sand curves since RCTS testing of the saprolite was not complete at the time the analysis was needed. RCTS tests were performed on three samples of the in-situ saprolite. The results of one test were included in the initial version of the VCSNS FSAR (sample UD2 from boring B-309, Figures 2.5.4-218a and 2.5.4-240a).

The results of the two RCTS tests on in-situ saprolite not included in the VCSNS FSAR are included here as Figure 02.05.04-2-1 (sample UD4 from boring B-325) and Figure 02.05.04-2-2 (sample UD3 from boring B-208). The results from the two saprolite samples consisting mainly of silty sand (sample UD2 from boring B-309 [Figure 2.5.4-240a] and sample UD4 from boring B-325 [Figure 02.05.04-2-1]) are similar and the generic EPRI sand curve used for G/G_{max} versus shear strain in the SHAKE analysis is close to a best fit curve for the RCTS G/G_{max} results for both samples. The generic EPRI sand D versus shear strain curves agrees well with the torsional shear RCTS results for both samples. The torsional shear results are lower (more conservative) than the resonant column results at lower strains.

Although a large majority of the saprolitic soils consist of silty sands (about 69% according to FSAR Section 2.5.4.2.2.3) there are silt and silty clay saprolites distributed throughout the saprolite stratum. The more silty and clayey saprolites are due to a greater degree of weathering, and thus these soils are distributed unevenly and unpredictably since the degree of weathering depends on several different factors. For completeness, RCTS testing was performed on a sample of saprolite consisting of predominantly elastic silt (sample UD3 from boring B-208 [Figure 02.05.04-2-2]). As would be expected, the RCTS test results from this sample agree better with the generic

EPRI clay curves (the curves for plasticity index PI = 50 in this case) than the generic EPRI sand curve.

In summary, a SHAKE analysis was run using a saprolite soil column to obtain peak ground acceleration for use in an analysis to characterize the liquefaction potential of the saprolite, although the saprolite will be removed from the zone of loading influence of all the major plant structures. Saprolite consisting of silty sand makes up almost 70% of the saprolites onsite. The remaining silt and silty clay saprolites are randomly distributed throughout the saprolite stratum, and do not make up a discrete layer. They therefore could not be modeled in the SHAKE analysis. The generic EPRI sand curves used in the SHAKE analysis agree well with the results of the two RCTS tests performed on silty sand samples.



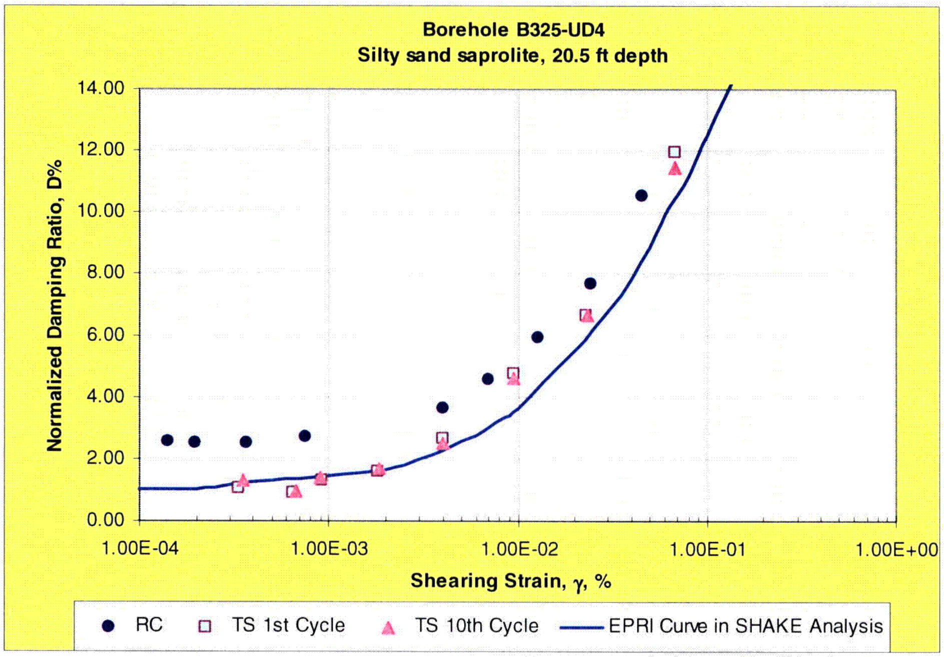
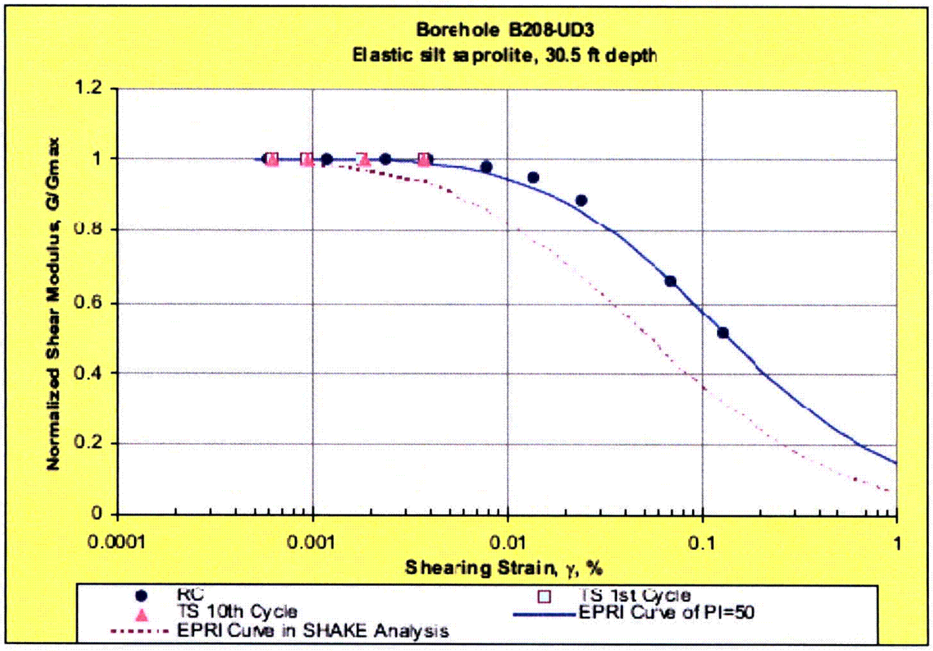


Figure 02.05.04-2-1



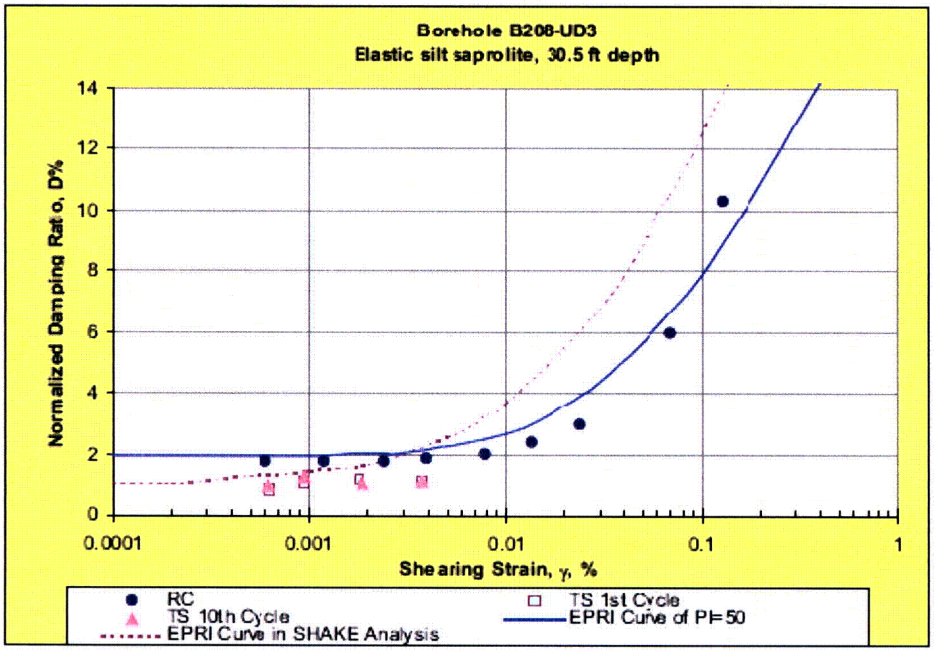


Figure 02.05.04-2-2

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

The results of the additional 2 RCTS tests (shown in Figures 02.05.04-2-1 and 02.05.04-2-2 will be included in a future revision of the COLA.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 10, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-5

FSAR Section 2.5.4.5.3.1 (pg 2.5.4-23) states that placement and compaction of structural backfill will be in accordance with technical specifications and procedures and in-place density testing frequency (e.g., a minimum of one per 10,000 square feet of fill).

In order for the staff to assess the adequacy of the structural fill, please provide the following information about structural fill placement and compaction:

(a) A justifiable testing frequency for performing field density testing for VCS Units 2 and 3 structural backfill.
(b) The basis for selecting this density testing frequency.
(c) How the frequency and number of density tests will provide assurance of adequate uniformity of shear wave velocity as required by the AP1000 standard design.

VCSNS RESPONSE:

(a) & (b) For compacted structural fill, lift thickness will be limited to ensure that the required degree of compaction is obtained. Optimum lift thickness and compactive effort will be based on the results of a test fill program per FSAR Subsection 2.5.4.5.3.1. An 8 in. to 12 in. loose lift thickness is anticipated. For one test every 10,000 square ft area, this is equivalent to one test approximately every 250 to 370 cubic yd. Table 5.6 of ASME NQA-1-1994 (ASME 1994) provides a listing of various field density testing frequencies depending on the circumstances of the fill placement. The most stringent requirement listed is one field test every 200 to 300 cubic yards of fill placed. The FSAR will be changed to state that compacted structural fill placement and testing will follow the guidelines of ASME NQA-1-1994, and that at least one field density test will be performed per lift and per shift, and for no more than every 250 cubic yards of fill placed.

(c) The response to RAI 02.05.04-1 described in some detail the proposed material to be used for structural fill at the site. (Note that this fill will not be used beneath the seismic Category I nuclear islands – concrete fill will be used as needed.) The structural fill is produced from crushing operations at a granite quarry, and as such, is consistently a well-graded, medium to coarse sand with little fines that will result in a total unit weight of about 130 pcf when compacted to 95% of modified Proctor compaction. It is anticipated that this fill will provide a more consistent level of field density than sands obtained from most in-situ sources. As a result, the one field density test per 250 cubic yards (within the most stringent ASME NQA-1-1994 requirement range) is considered conservative. The combination of very consistent structural fill material and frequency of field testing should ensure an adequate uniformity of shear wave velocity results, bearing in mind that the shear wave velocity of the compacted fill will be a function of confining pressure, and will thus increase with increasing depth.

References:

ASME (1994). American Society of Mechanical Engineers, ASME NQA-1-1994, Quality Assurance Requirements for Nuclear Facility Applications, New York, NY, 1994.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

The last paragraph of FSAR Subsection 2.5.4.5.3.1 will be revised as follows:

Fill placement and compaction control procedures are addressed in a technical specification. It includes requirements for suitable fill, sufficient testing to address potential material variations, and in-place density testing frequency. ~~(e.g., a minimum of one test per 10,000 square feet of fill placed). It~~ The compacted structural fill placement and testing follow the guidelines of ASME NQA-1-1994 (Reference 253), with one field density test being performed per lift and per shift, and for no more than every 250 cubic yards of fill placed (Table 5.6 of Reference 253). The specification also includes requirements for an onsite testing laboratory for quality control (e.g., gradation, moisture density, placement, and compaction) and requirements to ensure that the fill operations conform to the earthwork specification. The soil testing firm is required to be independent of the earthwork contractor and to have an approved quality program. Sufficient laboratory compaction (modified Proctor) and grain size distribution tests are performed to ensure that variations in the fill material are accounted for. A test fill program is also included for the purposes of determining an optimum size of roller, number of passes, lift thickness, and other relevant data for achievement of the specified compaction.

The following reference will be added to the Section 2.5.4 references:

253. American Society of Mechanical Engineers, ASME NQA-1-1994, *Quality Assurance Requirements for Nuclear Facility Applications*, New York, NY, 1994.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 10, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-7

FSAR Subsection 2.5.4.7.1.2 (pg 2.5.4-27) states that there are no measured shear wave velocity measurements for structural backfill because none has been placed, and a profile of the fill was developed using assumptions as discussed in Subsection 2.5.4.2.5.3.

In order for the staff to evaluate the adequacy of the fill profile and the assumptions used, please discuss how shear wave velocity testing of the compacted structural backfill will be performed, including its interface with any placed concrete backfill, and what assurance will be provided that the resultant shear wave velocities will meet the velocity requirements of the AP1000 standard design for the completed as-built condition.

VCSNS RESPONSE:

There will be no compacted granular structural fill placed beneath the seismic Category I nuclear island. Thus, shear wave velocity testing of the compacted structural backfill is not anticipated, and there are no required shear wave velocities for the completed as-built condition.

The relative concrete fill and structural fill locations are shown on FSAR Figures 2.5.4-220 through 2.5.4-223. The concrete fill will extend a few feet out from beyond the footprint of the nuclear island, where needed, and structural fill will be placed above it. This is the only situation where there will be any contact between the two types of fill.

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None

NRC RAI Letter No. 031 Dated February 10, 2009

SRP Section: 2.5.4 – Stability of Subsurface Materials and Foundations

Question from Geosciences and Geotechnical Engineering Branch 1 (RGS1)

NRC RAI Number: 02.05.04-10

FSAR Figure 2.5.4-224 shows that the shear wave velocity is variable across the power block area (PBA). Regulatory Guide 1.132 states that, where variable conditions are found, spacing of boreholes should be smaller, as needed, to obtain a clear picture of soil or rock properties and their variability.

In order for the staff to determine whether more boreholes may be needed, please provide justification for not having a smaller spacing of borings within the Unit 2 PBA and Nuclear Island (NI) footprint to better define the rock quality designation (RQD) and shear wave velocities.

VCSNS RESPONSE:

FSAR Figure 2.5.4-211 shows the RQD values from 15 Unit 2 borings in the Layer V sound rock. Six of the borings are beneath the nuclear island, and the majority of the remainder are from beneath the annex and turbine buildings. The RQD values are averaged for 5 ft intervals. From El. 320 ft down to El. 75 ft, these average RQD values are all between 90% and 100%. From El. 320 ft up to the proposed base of the nuclear island at El. 360 ft, the average RQDs are between 80% and 90%. Attached are the percent recovery values for the same 15 borings (Figure 02.05.04-10-1). From El. 300 ft down to El. 75 ft, all of the recovery values are 100%. From El. 300 ft up to the proposed base of the nuclear island at El. 360 ft, the average recoveries are between 90% and 100%, with the majority over 95%. These are by no means variable conditions; therefore a closer boring spacing is not required.

FSAR Figure 2.5.4-224 shows the measured shear wave velocities in four Unit 2 boreholes. FSAR Figure 2.5.4-226 shows theses data reduced to average and +/- 1 standard deviation. From El. 300 ft down to El. 90 ft, the average shear wave velocity is close to 10,000 fps, and the maximum standard deviation is less than 500 fps (coefficient of variation, COV = 0.05). From El. 300 ft to El. 325 ft, the average is closer to 9,500 fps and the maximum standard deviation is just over 1,000 fps (COV = 0.11). Between El. 325 ft and El. 355 ft, the average velocity drops to just over 8,500 fps and the maximum standard deviation is just under 2,000 fps (COV = 0.23). The increase in variability towards the rock surface is expected – although this is among some of the strongest and least fractured/weathered rock encountered in the Piedmont, some degradation towards the surface is almost inevitable (except under Unit 3 which is truly exceptional). Again, for a bedrock environment, these are by no means variable conditions; therefore a closer boring spacing is not required.

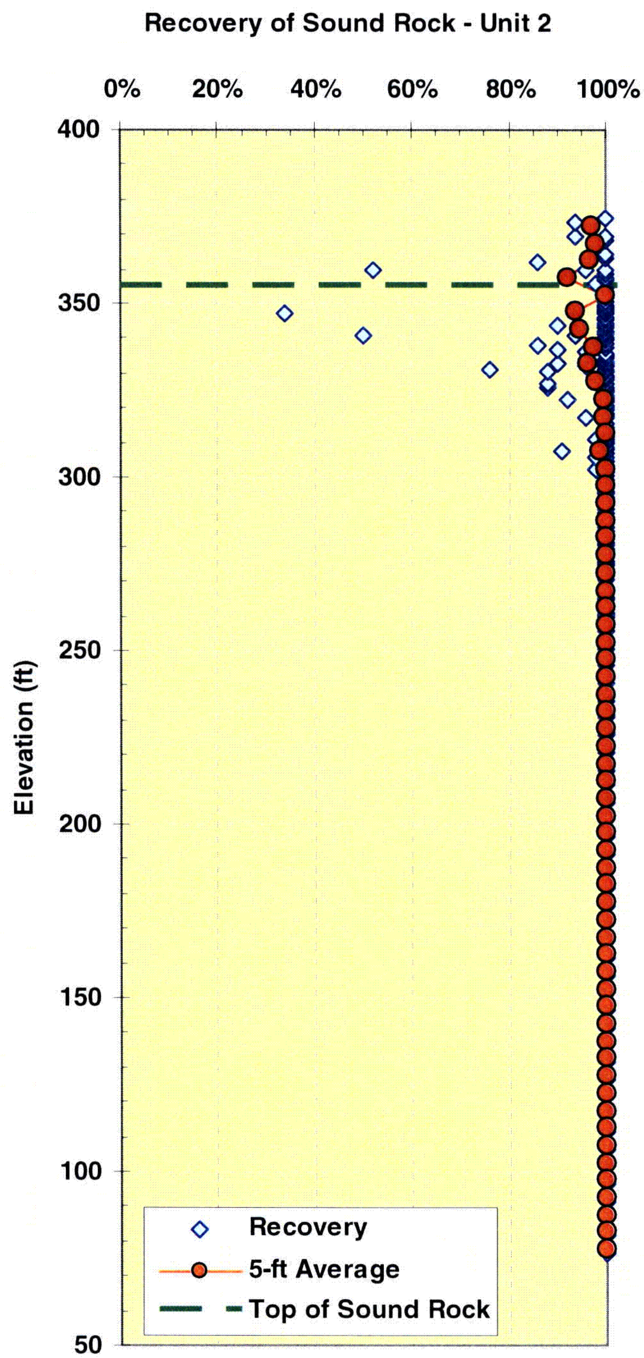


Figure 02.05.04-10-1

This response is PLANT SPECIFIC.

ASSOCIATED VCSNS COLA REVISIONS:

No COLA changes have been identified as a result of this response.

ASSOCIATED ATTACHMENTS:

None